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STUDIES OF BEARPAW SHALE AT A DAMSITE IN SASKATCHEWAN

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STUDIES OF THE BEARPAW SHALE AT A DAMSITE IN SASKATCHEWAN

Robert Peterson,[1] M. ASCE

SYNOPSIS

This paper describes the soil mechanics and geological investigations which have been undertaken to study the Bearpaw shale, a member of the clay-shale group, at a proposed damsite in the Province of Saskatchewan. In the course of the investigations the geology was reviewed in an attempt to clarify the geological history of the immediate vicinity. In addition, extensive use was made of air photo analysis in locating slides in the shale. The investigations included field observations of the shale on natural slopes, from test holes, in deep shafts and in a test drift, as well as an extensive program of laboratory testing. The studies have revealed the characteristics of the shale and have indicated the location and extent of slides which might have an effect on the proposed construction. However, there is still considerable scope for speculation as to the sequence and nature of the geological processes that formed the region.

Introduction

Large areas of Western Canada and the North Central States are underlain by clay-shale types of bedrock which, in the past, have been the cause of foundation and stability problems because their physical characteristics were not completely understood. Geologically, these formations have been mapped and well described. However, it was not until the Prairie Farm Rehabilitation Administration(1) began detailed studies for an earth dam on the South Saskatchewan River that a concerted effort was made in Canada to study the physical properties of this unusual material.

In the early search for a damsite, the Bearpaw shale was regarded as a good foundation material. Geologically it was classed as bedrock although many engineers preferred to regard it as a hard clay. In the undisturbed state this material was dense, homogeneous and impervious and also appeared to possess sufficient strength to support the required structures. However, detailed studies revealed certain undesirable characteristics such as the tendency to swell and soften when subjected to weathering. The surface material, at least, was unstable. Subsequent studies confirmed the original belief that the harder shale was competent to support a high earth dam. However, because of the tendency of the shale to swell after the removal of load it was obvious that

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careful design would be necessary to avoid heave where deep excavations were required for structures. It was also concluded that while the harder shale would be stable on relatively steep slopes for a short period, much flatter slopes would be necessary for long time stability. This fact has exerted a profound influence on the choice of site and layout. It has dictated flat slopes on the dam cross-section and has indicated the desirability of flat abutment slopes and shallow cuts adjacent to structures. To illustrate the importance of this limitation: initially it was hoped that a 150 foot cut in the shale would be stable on 3:1 slopes; it is now believed that this depth might require 6:1 or 8:1 slopes for long time stability. While these slopes may seem, and may very well be, ultra-conservative, such conservative designs are justified on the basis of the available evidence.

Scope of Studies

Commencing in 1944, foundation investigations, surveys, and general studies(2) were carried out at ten prospective damsites between points A and C, Figure 1. In general, foundation conditions were similar but other requirements indicated that the stretch of river between B and C would be the most desirable site. The site which has been chosen provides for an excellent spillway location remote from the dam, and ample sources of fill material and concrete aggregate are easily available.

At the site, shown in Figure 2, Coteau Creek and Hudson's Bay Coulee provide a natural route for a spillway with a control structure site near the junction of the two. This layout permits the earth dam, diversion works and power house to be located at the most suitable point in the two mile stretch of river between points E and F. The dam as now proposed involves an earth fill 205 feet in height.

In planning the preliminary investigation at this site the services of a Geological Consultant(3) and an engineer trained in air photo interpretation(4) were utilized. In this phase of the investigation 2-1/2 inch drive samples were recovered at 5 foot intervals in the drill holes. In addition, some 2 inch and 6 inch core holes, a few test pits, and several large diameter auger holes were put down. The main object was to locate the shale surface and to recover samples from which the properties of the various materials could be determined.

A study of the results of the preliminary investigations, and experience elsewhere, indicated that the shale would present unusual foundation and stability problems. At this stage of the investigation A. Casagrande of Harvard University consented to become Soil Mechanics and Foundation Consultant. His experience with clays and particularly with clay-shales has been most valuable and his direction of the second phase of the investigation has resulted in a somewhat different approach.

In the second phase of the investigation, emphasis was placed on field observations of the shale coupled with careful laboratory testing of a few select samples. Based on experience elsewhere with brittle clay and clay-shale it seemed apparent that the application of shear test results in stability studies did not always check with field observations of slope stability. It was therefore concluded that a program of

field observations would likely yield reliable information on the characteristics and behaviour of the shale. For a better understanding of the geological forces that had produced slides and instability in the shale it was necessary to review in greater detail the geology of the general region and the immediate site. The air photos were re-examined to locate and study all unstable areas in the vicinity of the site and in other typical areas along the river. A test drift, deep test pits and test holes with continuous cores were also utilized at this stage. Further information was obtained from a pressure test section in the drift which was designed to measure the stresses exerted by the shale on a continuous lining. Observations were made to determine the piezometric pressure in the shale, and to detect areas of active movement by establishing alignment and elevation plugs on the slopes. Extensive water content and Atterberg limit tests were carried out for purposes of correlation and comparison. Further shear and consolidation tests were performed with the hope that predictions based on laboratory tests would be in agreement with field behaviour.

Regional Geology

Most of the area shown in Figure 1 has been glaciated with the result that the bedrock is generally covered with a mantle of drift except along the deeper valleys where outcrops occur. The hatched areas on Figure 1 are underlain by soft sedimentary rocks of Upper Cretaceous age herein referred to as clay-shales. Three separate symbols are shown to differentiate the Bearpaw Formation, the Pierre Formation, and a group called "Other Clay-Shales." This latter group includes several formations each having a different name in the Provinces of Western Canada but all apparently equivalent to the Pierre Formation in the United States.

Geologically[5] the groups referred to above have a number of characteristics in common. They are of marine origin and were deposited in the Pierre sea which extended from the Arctic to the Gulf of Mexico. These shales are very uniform and are usually medium grey in color. They are soft, slake easily with water and on moistening expand to form a highly plastic clay. The shales contain a high percentage of montmorillonite and in some instances layers of bentonite are present. On weathered surfaces, crystals of gypsum are often found. In addition, the shales frequently contain pyrite which on oxidation gives an acid reaction.

It has been estimated[3] that sediment, at least 2000 feet and possibly 2500 feet in thickness, has been eroded from the area of the damsite. This is based, in part, on the fact that the strata are nearly flat lying and that sedimentary deposits of more recent origin than the sediments at the damsite have been found at point D, Figure 1, which is approximately 3000 feet higher than the damsite. The shale has therefore been subjected to a large reduction in confining pressure and is tending to swell or expand.

It is believed by geologists that the valley of the South Saskatchewan River between points A and C was cut by one or more of the four

different advances of the ice sheet. In preglacial times this drainage existed but it is thought that it was located to the west and north of the present channel between A and C. At the damsite it is believed that the valley was carved out of the shale by the second or third advance of the ice.

Geology of the Damsite

The area in the immediate vicinity of the damsite is shown in Figure 2. The river channel is about 2000 feet wide although this channel is partially covered by sandbars at low stages. The present valley is approximately 200 feet deep and the banks, from the rim of the upland to the river's edge, are on slopes of the order of 8:1 to 12:1.

The river channel is underlain by fine to coarse sand. At the edges of the channel the sand is shallow and the shale is close to the surface but near the center approximately 100 feet of sand overlies shale. Profiles across the river valley along lines R-1 and R-6, Figure 3, show the materials encountered in the streambed and on the abutments. In general, glacial clay overlies the shale on the right (east) valley slope, whereas sand and silt overlie the shale on the left (west) side. The entire plateau area west of the river, enclosed by Coteau Creek and Hudson's Bay Coulee, consists of nearly 100 feet of sand and gravel over the shale bedrock. This material apparently was deposited as outwash during the period when the retreating ice sheet blocked the river flow to the north and drainage from Coteau Creek was south to the Missouri Basin.

Slumping is common along the South Saskatchewan River valley[3] and in the vicinity of the damsite. On the basis of aerial studies,[4] areas of definite movement have been indicated in Figure 2 by full lines and areas where movement might possibly have occurred by dashed lines. It will be noted that movement is indicated on the west slope at line R-1 whereas there is only a possibility of the movement in this area at line R-6. The photographs shown in Figure 2, 2A, 2B, 2C, and 2D indicate that there is a characteristic difference between the two areas. The R-1 area (Figure 2A and 2B) is hummocky and contains arc-like ridges which are generally parallel to the river. Undrained pockets containing vegetation and brush are frequent. The most pronounced ridge occurs adjacent to the upland and in this case it would appear that the material making up the ridge has moved out leaving an arc-like trough about 20 feet in depth. It is significant to note that a corresponding depression occurs in the underlying bedrock and that the top of the ridge is nearly as high as the plateau, indicating the probability of horizontal movement. Of further importance is the occurrence of several cross dykes in the bottom of the trough at right angles to it. These dykes raise the possibility of movement having occurred before the last retreat of the ice. In any case, the smoothness of the hummocks and ridges would indicate that the movement was very ancient. There was some doubt as to whether movement occurred on the shale overburden contact, through the soft and weathered shale, or in the hard shale. In contrast to the scarred and hummocky topography on the west

bank on line R-1 the ground surface at R-6 is smooth, unscarred and uniform in slope and appearance (Figure 2C and 2D). It is now grassed and little or no brush occurs, likely because the area has been under cultivation. Based on the test borings and surface indications it would appear very unlikely that movement had occurred in this area.

It was obvious that movement had occurred on the west bank at line R-1 but it was unlikely that movement had occurred at R-6. This information resulted in the area downstream from line R-6 being favoured as a centerline for the proposed dam with the outlet conduits located in the shale bench west of the river channel.

Field Studies of the Shale

The field and laboratory program described in the following sections was carried out in the second phase of the investigation.

1) Test Drift

When it became evident that movement had occurred on the west bank in the R-1 area it was decided to excavate a test drift into the shale in order to assist in studying the extent and nature of the movement. In addition, such an opening provided an opportunity to examine the shale and to obtain data on mining and handling methods.

The test drift, located on line R-1 a few feet above river level, was 365 feet in length, 6 feet wide, and 8 feet in height. The shale was excavated easily using air spades and was mucked into hand cars. Timber bents consisting of 8 inch by 8 inch timbers were initially placed on 4 foot centers, this being the maximum distance that could be mined without support. However, as the drift progressed it became necessary to decrease the spacing depending on local conditions. The roof was lagged continuously and the sides intermittently as required.

A comprehensive sampling program of the shale was undertaken along with numerous observations to detect movement in the shale and the supporting timbers. Shale samples were taken at close intervals for water content and Atterberg limit determinations. Chunk samples were also taken for testing purposes. Test holes were drilled from the ground surface to drift level, and also horizontally in both directions from the drift. In addition, pits were put down from the drift floor. Plugs were set in the shale between bents in both the floor and walls to detect any upheaval of the floor or tendency for the shale walls to move inward. Observations were also made to detect any rotation or deflection of the bents.

Fortunately the drift was located so that a movement plane in the shale was intercepted. This involved soft disturbed shale moving on a flat arc-like plane over the underlying hard shale in a direction parallel to the drift toward the river. The floor of the drift was in the hard shale throughout its length and the movement plane varied in height above the floor, but rose to the top of the drift at a distance of 260 feet from the portal. A rotation of the timber bents toward the portal during excavation of the drift indicated a displacement of the order of 2 inches.

It is believed that this movement occurred on an old slide plane and that it was reactivated by a steepening of the ground slope to obtain a suitable face from which to start the drift. The soft shale in the upper portions of the drift moved into the space between bents a distance of from 11 to 21 inches (i. e., total width decreased 11-21 inches). A large number of timber uprights failed in bending due to pressure exerted by the soft shale. These were removed and new bents placed at closer intervals. In comparison, the hard shale moved into the opening at a distance of only 1 to 3 inches and floor plugs moved vertically 0.3 inches. A large portion of the movements occurred within a matter of a few weeks but the totals given are for a period of 1-1/2 years after construction.

A special effort was made to choose a simple test which would indicate the general character of the shale. Of the water content, density, and Atterberg limit tests, it was concluded that the water content was the easiest to perform and probably the most significant indicator of the condition of the shale. The water content profile shown in Figure 4 illustrates the sharp break associated with the change from the soft to the hard shale zone which in this case is the movement plane referred to above. It should be noted, however, that in some instances changes in water content may be associated with a change in the plasticity characteristics — particularly the liquid limit.

2) Continuous Cores

Experience in the test drift indicated that accurate water content profiles are useful in establishing the boundaries between consistency zones in the shale and possibly in locating sliding surfaces. In order to avoid any possible increase in water content by conventional coring methods continuous drive samples were recovered from dry holes using churn drill equipment. This method has proven very satisfactory. After determining the density of individual samples, they were classified and examined to detect the presence of joints, slickensides, silt or bentonitic lenses, or any other unusual features. In addition, samples were taken for water content and Atterberg limit determinations, generally at 3 inch or 6 inch intervals. The index properties consisting of natural water content, plastic limit, liquid limit, relative consistency, and density were then plotted. The relative consistency is defined as the liquid limit minus the natural water content divided by the plasticity index.

A correlation between the index properties and the observed consistency and strength of undisturbed samples has established a number of consistency zones in the shale, described as soft, medium, and hard, occurring in order from the surface downward. However, in some areas the soft or medium zones may not be present. The water content apparently is the best indicator of consistency and it generally decreases with depth from the shale surface except where drying has reversed the trend near the surface. The water content of the soft zone is frequently very erratic whereas the water content of the medium and hard zones is more uniform. The water content at the surface of the hard zone is about 25 to 27 percent and its average decrease is approximately 7 percent in a depth of 100 feet.

3) Deep Test Pits

To supplement the information obtained by continuous cores and in an attempt to correlate movement planes with the indicators used in studying the cores, a number of deep test pits were excavated. These pits also provided an opportunity to examine variations in the shale and to obtain undisturbed samples for testing. Large diameter uncribbed auger holes had proven unsafe because of caving in the weathered shale and it was therefore necessary to use a lining which would provide support and also permit examination of the material. The method developed consisted of continuous wooden cribbing through the overburden and alternate rings of steel liner plate, and wire mesh supported by spacers, through the shale as shown in Figure 5. Pits up to a maximum depth of 160 feet were dug although numerous safety precautions were necessary.

The open pits provided an excellent opportunity to examine the shale in place and to study the consistency zones, slickensides and joints. Slickensides and joints occurred in all directions and had no relation to slide surfaces. The slickensides varied from mirror-like surfaces with no sign of movement to grooved and fluted surfaces containing heavy striations. Their size varied from less than one square inch to many square feet in area. Quite frequently very abrupt curvature in the direction of the striations was evident. In areas of intense slickensiding a large chunk bounded by slickensides could often be broken down into many smaller chunks, each bounded by a slickenside. In most pits the intensity of slickensiding decreased with depth.

It proved much more difficult to locate movement planes in the pits than would be expected. The continuous core holes and observations in the drift strongly suggested that in stable areas the water content decreased uniformly with depth through the various zones (with some local variations) whereas in unstable areas the water content appeared to decrease abruptly at the lower boundary of the movement. This coincided with the bottom of the soft zone. Conclusive evidence of a slide plane was found in only one pit. However, in several other pits, located in areas where movement had occurred, zones of badly broken and disturbed shale were found and these were probably produced by the slides. The broken and disturbed shale had high water contents and it is believed that its lower boundary coincided with the lower limit of movement.

4) Pressure Test Section

At the end of the test drift a special section was constructed to measure the existing horizontal and vertical pressures in the shale. The transition from the drift to the pressure test section consisted of a heavily timbered section 20 feet long. The pressure test section was also 20 feet in length and was lined with precast concrete slabs as shown in Figure 6. The slabs were 2 feet wide by 8 inches thick and approximately 6 feet long. The horizontal and vertical slabs were staggered so that the joint between horizontal slabs fell at the center of the vertical slabs. The slabs were held in place by pipe struts on the ends of which were fastened screw jacks. Two horizontal struts were used to hold two vertical slabs and two vertical struts to hold two horizontal slabs. The excavation and the placing of the slabs was carried out

continuously over a period of 10 days. Dry packed grout was used to back pack the irregular space between the slabs and the mined surface of the shale. When the installation was completed a predetermined load was applied by the screw jacks which reacted through the pipe columns to the slabs. Initially a load approximately equal to 130 percent of the overburden stress (overburden above drift - 64 feet) was applied to both the horizontal and vertical slabs. After the final adjustment was obtained load determinations were taken periodically. The load decreased on both the vertical and horizontal slabs for a period of about three months and then began to increase very slightly. Two years after installation the load on the horizontal slabs was approximately equal to overburden and the load on the vertical slabs was approximately equal to the initial applied load. It would therefore appear, as was originally suspected, that the horizontal pressure exerted by this preconsolidated shale is greater than the vertical and that such a pressure ratio might be expected upon a tunnel lining.

The load on each strut was determined by means of strain measurements utilizing a 20 inch Berry strain gauge. Initially all 40 struts were calibrated in the laboratory and load-deformation curves determined. It is believed that the load determinations on the struts are within 10 percent of the true values.

5) Piezometer and Water Level Observations

The early borings to the Belly River Formation which underlies the Bearpaw shale, as indicated in Figure 3, revealed that the sandstones in this formation contained water under pressure. Initial determinations indicated this head to be approximately 70 - 100 feet above river level. Wells within a distance of 10 - 15 miles indicated that the Belly River sandstones extended over a considerable area and that the water level was of the order of 200 - 300 feet above river level. In view of these facts, it was suspected that the Bearpaw shale was under excess hydrostatic pressure, and a number of piezometers were installed in the R-1 area on the west abutment. Each piezometer[6] utilized a tip consisting of a porous ceramic tube about one inch in diameter and twelve inches in length which was connected to a standpipe of one half inch saran tubing. In general, the porous tips were located about 45 or 50 feet below the shale surface. Two years of readings indicated that there was no excess pressure in the shale at this depth and that the piezometric level varied from 20 - 30 feet below the shale surface as shown in Figure 3.

Drill holes and standpipes into the surface of the shale indicated that the ground water table, as shown in Figure 3, was close to the shale-overburden contact. The observations in standpipes and drill holes checked with the data obtained in the pits where the water level was usually encountered in the weathered shale and some seepage drained into the pit from this zone. The underlying unweathered shale was impermeable and little or no seepage occurred even at considerable depths below river level.

6) Slide Studies

In analyzing slides along the South Saskatchewan River one important question was whether the deep glacial trough in the shale beneath the riverbed had been empty. During the early stages of the investigation it was believed that subsequent to erosion by ice, this trough had always been partially filled with glacial ice and debris. This hypothesis checked with the belief that the slides along the river did not extend far below the present level of the river channel. More recently undisturbed samples of the river sand established that the entire depth of sand was river deposited. A profile of the shale line at right angles to Coteau Creek, near the river, revealed that the low point on the profile was about elevation 1595, i.e. 50 feet below present river level and 50 feet above the bottom of the deep trough. This would tend to substantiate the conclusion that the river at one time flowed near the bottom of the trough and has been aggrading since that time. It therefore became evident that an opportunity for slides into the deeper channel existed in the past. The borings were re-examined in an attempt to locate evidence of slides into the deep trough. In only one area has evidence indicated such a slide and this is shown on the profile of Line R-6 in Figure 3. This slide occurred on the east bank and apparently extended down to elevation 1580. The overthrust has been traced by 4 drill holes from a point 1100 feet upstream from R-6 to a point 1400 feet downstream from R-6. It is therefore probable that the slides along the river extend to some depth below present river level.

When plans were first made to study existing slides in the shale it had been hoped that several slides at some distance from the site could be analyzed in detail. However it soon became evident that a prohibitive amount of exploration would be required to establish the soil profiles and locate the slide surfaces. In view of this the slides in the vicinity of the site which had been drilled were considered in detail and the more remote slide areas only superficially.

Of the slides within the site area, the west bank at line R-1 has been considered in detail. The soil profile, piezometric pressures, ground water levels, and other pertinent data have been given in previous sections and are summarized in Figure 3. In addition, a number of observation hubs have been checked for alignment and elevation and no apparent movement has occurred during a two year period. On the basis of topographic indications it is quite obvious that movement or creep has occurred in the past. It would appear that the movement has been slow and that it has occurred through the soft surface layer of the shale parallel to the slope rather than as a deep seated slide through the hard shale. The movement is believed to be very old, possibly having occurred before the retreat of the last ice sheet. Further detailed studies of the cross dykes, referred to previously, seem to support this possibility.

These dykes are located at intervals along the depression which intersects line R-1 near Station 0+00. They are spaced as shown in Figure 2 and the axis of each dyke is parallel to line R-1 and at right angles to the general direction of the depression. They are very regular in form and are generally about 3 - 4 feet in height at the maximum

section. Some have flat tops, 2 - 3 feet wide, whereas others come almost to a point with side slopes from 2:1 to 3:1. For any one dyke the cross section is fairly constant and the top generally slopes slightly toward the river. Adjacent to both ends of each dyke there is a depression in the terrain very similar to that found above a glacial fan. The material in the dykes is uniform consisting of well graded sand and fine gravel with silt and clay binder. Immediately beneath the dykes is a 3-4 foot layer of sandy silt and silty sand which is underlain by clean sand and gravel. There is slight evidence of water sorting and organic layers, within the dykes. The first impression upon inspection of these dykes is that they were man made, however, this seems improbable. Therefore it is tentatively suggested that they were deposited during the last stage of glaciation and that the movement in the underlying shale occurred previous to that time.

In studying other slide areas along the river, extensive use was made of air photo interpretation[4] and ground observations. A number of profiles were syrveyed and staked in order to make stability computations and to detect any movement. In some instances a few shallow pits and auger holes were dug.

In general, the topography in other slide areas was very similar to that at R-1. The areas were very hummocky and frequently consisted of crescent shaped depressions and ridges, generally parallel to the river. In cases where till or aluvium occurred above the shale it appeared that the movement had been mainly horizontal. It seemed that there was a tendency for the underlying shale to yield, forming cracks in the overburden which finally widened and resulted in subsidence blocks similar to that shown in Figure 7. It would appear that many of the slides have occurred as a result of river bank erosion. Slides then progressed from the river toward the upland. It is believed that the slides occurred slowly and intermittently over a period of many hundreds of years. This to some degree is supported by the fact that the hummocks adjacent to the river were more rounded than those close to the upland. Initially it was thought that all the movements were postglacial, however, there is a possibility of interglacial movement. Based on the surveyed lines there has been no indication of active movement over a period of several years. However, in some instances there has been fairly recent secondary movement immediately adjacent to steep cut banks near the top of old slides. In one area a slide has occurred on a steep shale slope since the original aerial photographs were taken in 1935. While the old movements have generally been referred to as slides, they were probably a combination of sliding and creeping. Creep is defined by Terzaghi[7] as a "continuous movement which proceeds at an average rate of less than one foot per decade."

A general examination of the aerial photographs has indicated several smooth stable areas similar to R-6. In two of these cases the orientation is similar to that at the site in that the stable topography lies between the river and a drainage channel entering from the west. The presence of these stable areas may be due in part to the better drainage which is provided in two directions or to the flat slopes to which the shale has been eroded by glaciation.

An examination of many of the gullies entering the river has indicated slumping or sliding very similar to that which has occurred along the river. In the case of these gullies it is known that shale is near the surface in the bottom. However, in spite of this, the general slide topography in these gullies and along the river is very similar. This suggests that the slides along the river were not deep seated in the shale.

Stability computations have been carried out for R-1 and several other slides. In the case of the latter, the computations were based only on surface features with assumptions being made regarding the location of the sliding surfaces, soil densities and water levels. In view of these unknowns the results are not included. In the R-1 area more complete information was available and a flat arc was assumed through the soft shale from the slide surface in the test drift to the shale depression near the upland. The shear strength based on these assumptions and for a factor of safety of 1.0 was 15 pounds per square inch.

Laboratory Studies of the Shale

1) General

Standard laboratory procedures were used, where applicable, to test the shale as it is no more than a hard clay. However, special procedures have been required for classification, Atterberg limits, strength-deformation tests and consolidation tests.

2) Summary of Soil Test Results

There are generally three zones of consistency in the shale formation, that is, soft, medium and hard. Many of the test results vary according to the consistency zone as shown in Table No. 1. The results given are for typical waxy shale in the unweathered condition. Where weathering and oxidation has occurred the properties may vary considerably and the entire mass may be composed of chunks bounded by joints having a characteristic brown coloring. One of the most pronounced changes due to weathering is increased permeability because water travels easily along the oxidized joint planes.

3) Method of Classification

At the outset it was proposed to describe the shale and other bedrock materials in accordance with the Casagrande[8] system of soil classification. This system utilizes the plasticity characteristics to describe fine grained soils and the grain size distribution for coarse grained soils. This method was not intended for classifying bedrock and does not include the words shale or sandstone, but in view of the fact that they were thought to behave like clay and sand respectively it was hoped that no inconsistencies would be encountered.

Based on visual examination the majority of the shale samples were classed as waxy shale with the more plastic ones being designated as bentonitic shale and the less plastic samples as silty or sandy shale.

The first inconsistency regarding classification occurred in connection with a very sandy layer in the Bearpaw Formation which was classified as soft sandstone by Geologists and field inspectors. However, the Atterberg limits of this sandstone indicated that it was medium to highly plastic and therefore vastly different in character than sands as defined by the Casagrande system. Based on Atterberg limits its plasticity approached that of the shale but its appearance and feel was that of uncemented sandstone. If therefore, the Casagrande system was to be followed, this material should be classed as a shale or clay. This would not be consistent with the geological classification based on appearance and feel. In view of this it was decided to retain a geological classification for all bedrock materials with the descriptive adjectives being based on feel and appearance rather than plasticity characteristics. This provided an acceptable means of description for classification purposes. However, plasticity characteristics were used in design studies as it was found that the actual properties based on permeability, shear and consolidation tests correlated better with plasticity characteristics than with the geological classification.

4) Atterberg Limits

It is now recognized that the Atterberg limits are the most significant routine tests which can be performed on cohesive soils. In view of this the limits were used to study variations in the shale, indicate the general character of the material and make comparisons with other materials. Liquid limit tests carried out at Harvard University[9] indicated that the same shale sample prepared by soaking from air dried, oven dried or natural water content conditions yielded vastly different results. It was found that oven drying decreased the liquid limit and air drying increased the liquid limit considerably from that obtained by using the material at its natural water content. The following liquid limit values were obtained from one sample by varying the method of preparation: undried - 132, air dried - 163, and oven dried - 99. It was found further that the values obtained from undried specimens varied considerably with soaking time and mixing time. In contrast the values for oven dried and air dried were more consistent as the material could be easily mixed into a homogeneous paste. While it was believed that the liquid limit based on undried specimens was the most significant it was concluded that the liquid and plastic limits based on air dried samples would be more practical for use on the shale.

The early tests using the air dried procedure indicated that the liquid limit in particular was very sensitive to minor variations in the procedure. In order to evaluate some of the factors affecting the results and to standardize a procedure, a short research program was carried out in connection with both liquid and plastic limit determinations.

The liquid limit program showed only minor variations due to: different operators, different liquid limit machines, weight of the liquid limit cup, change of pressure in placing the material in the cup, length of soaking time, and method of mixing. Major variations were caused by: sieve size of the material before soaking and mixing, the amount

of water used to soak the material, and the air dried water content of the material prior to soaking and mixing. Initially it was felt that the sieve size of the material before soaking affected the results because the soaking time was insufficient for the larger particles. However, experiments with increased soaking time resulted in no improvement and this variation remains unexplained. Further research is now being carried on to evaluate these variables and in the meantime an attempt is being made to keep them reasonably constant.

The plastic limit program investigated the variations due to different operators and different soaking periods. It was found that with reasonable care, making sure to roll the thread to the plastic limit or drier, the result was not very sensitive to the method of preparation.

The Atterberg limits for a large number of shale samples and a few sandstone samples have been plotted on the plasticity chart in Figure 8. It was believed that the shale was slightly organic and this was supported by the drop in liquid limit due to oven drying, this behaviour being associated with organic soils. However it should be noted that all the plotted values fall above the A line. Generally, inorganic soils are located above the A line and organic soils below the A line. In the case of the Bearpaw shale it would appear that the highly plastic properties have overshadowed the slight organic characteristics. Another point of interest is the fact that the limit values lie on a band which is inclined to the A line. Most soils of the same geological origin fall on lines very nearly parallel to the A line.

5) Strength — Deformation Tests

The strength-deformation characteristics of the three consistency zones of the shale were determined by means of unconfined compression tests, triaxial quick tests, and quick direct shear tests. The test results have been summarized in a preceding section.

The shear strength tests on samples from the medium and hard zones yielded satisfactory results, whereas it was almost impossible to prepare specimens from samples of the soft zone. These varied widely in water content and consistency, frequently containing many slickensides and joints. In view of this there is some doubt as to whether the test results for the soft shale are representative of this material in situ. Probably the most reliable value for the minimum shear strength of the soft zone was that obtained from the analysis of an actual slide within the soft zone as given under Slide Studies.

Except for minor variations, the shear strength and tangent modulus of the shale increased with depth. Excluding the bentonitic layers, the shear strength increased with a decrease in natural water content. A typical bentonitic sample had an unconfined compressive strength of 305 pounds per square inch at a water content of 48.3 percent.

An example of the excellent agreement between results of unconfined tests on specimens cut from the same sample of hard shale is shown in Table No. 2.

Investigations carried on at Harvard University[10, 11] revealed that the strength of some types of brittle clays and clay-shales under sustained load and at constant water content was less than the strength

indicated by unconfined compression tests. Their tests on the Bearpaw shale indicated that the strength might decrease to 80 percent of the normal value in 30 days. In view of this, a program involving creep-strength[10] and long time compression tests is now being carried out to study the variations in strength of the shale with the rate of loading.

6) Consolidation Tests

Consolidation tests were carried out on shale samples using both the swelling and non-swelling types of test. In the former, the specimen was allowed to swell during saturation, whereas in the latter, the specimen was maintained at its initial volume and the swelling pressure was measured. The results of the swell type were rather erratic, possibly because of disturbance due to swelling, and consequently this type of test was discontinued.

In general, specimens 4 inches in diameter and 3/4 inch in thickness were loaded, in floating rings, to pressures of 14 tons per square foot with a few specimens loaded to 27 and 58 tons per square foot. Even at these higher loads the pressure-void ratio line was still curved because these pressures were far below 100 - 150 tons per square foot, which is the pressure estimated to have consolidated the sediments. Each increment of load was allowed to act upon the specimen for a period of about 24 hours at which time the primary consolidation appeared to be complete. The average results for the consolidation tests are summarized in a previous section.

After analyzing consolidation tests of the type described above, Casagrande[9] concluded that the rebound portion of the pressure-void ratio curves from laboratory tests were much flatter than a pressure-void ratio curve based on void ratios and the corresponding overburden pressure of shale in situ. That is, for the same change in pressure a much larger change in void ratio occurred in place than the change in void ratio on the rebound portion of a laboratory curve. Theoretically the two curves might be expected to check as the shale is known to be in a state of rebound. It was suggested that the discrepancy might be due to the secondary time effect which had taken place over centuries in the field but which did not take place in a period of a few days in the laboratory test. As a result, studies have been started to determine the magnitude of the secondary time effect on both consolidation and rebound. In Figure 9 two time curves are shown covering a period of about 200 days. One time curve is for a 4 inch diameter specimen 3/4 inch thick loaded to 14 tons per square foot, which is approximately the unit stress beneath the proposed dam. It will be noted that the primary portion has been completed and that the compression in the secondary portion is already over 50 percent of the primary. The second curve is for swelling of a sample of similar dimensions under no load. Here again the secondary portion of the swelling is at least 50 percent of the primary.

Characteristics of Similar Shales at Other Projects

In conjunction with the studies being carried out for this project information on similar materials in other areas was compiled.

Literature dealing with the stiff fissured clays[12, 13] in southern England was reviewed although it is recognized that the two materials are somewhat different as the Bearpaw shale does not contain fissures. In addition, the experiences with the Cucaracha Formation at the Panama Canal[14] and the studies for the proposed sea level canal[15] were reviewed, as this formation is composed mainly of soft clay-shales.

The projects in the Missouri River Basin have been followed closely because clay-shales occur in portions of that region. The Bearpaw shale at the Fort Peck Dam[16] and the Pierre shale at the Oahe Dam[10] present problems similar to those under study. From the data available it would appear that the shale at Fort Peck and Oahe is somewhat harder; however, in both cases bentonite seams and faults are found which are not present in the local bearpaw Formation. The topography in the slide areas at Oahe is almost identical to that in slide areas in the vicinity of this site.

In Western Canada at least a dozen P.F.R.A. projects involving dams or canals have been built on Bearpaw shale although many of these were built before this program was started and therefore only limited data is available. Steps are now being taken to obtain observations on all structures and slopes in the Bearpaw shale.

Conclusions

Since the studies are still continuing, the following conclusions should be regarded as tentative.

The Bearpaw shale in past geological history has been compressed by sediments and ice exerting pressures of the order of 100 - 150 tons per square foot. In subsequent times this pressure has been removed by erosion and the shale is now in a state of rebound. In addition to vertical rebound the material is possibly yielding in the direction of the river channel. It has been found that slickensides and joint planes are prevalent in the shale although the frequency decreases with depth. In general, the slickensides are not associated with slide surfaces. However, there are several hypotheses as to their formation in clay-shales. It is believed by some that they are the result of chemical change; others state that they are the result of consolidation. Casagrande[9] has suggested that the slickensides have been formed during the rebound process, because at this stage the shale was more brittle and therefore sensitive to irregular strains and also because irregular erosion is more productive of large shear stresses than the period of sedimentation and consolidation.

Generally, soft, medium and hard consistency zones are present in the shale. The soft zone is very wet, containing numerous joint planes and slickensides. In addition to the rebound process some of the material in this zone has been subjected to weathering and movement, which

has changed the structure of the shale to that of a highly plastic clay. The underlying hard zone is quite uniform, sound and impervious, with only a limited number of slickensides and joints. The medium zone when present is a transition between the two. The tests have indicated that the hard zone is a satisfactory and competent foundation material if loaded so as to minimize rebound and if protected from weathering. The soft zone is an undesirable foundation material and old slides have occurred through it in some areas.

While no bentonite layers or marker beds have been located in the shale there are bands of bentonitic shale which, on the basis of shear tests, are as strong as the hard shale.

Where slides have occurred on relatively flat slopes the movement planes are believed to be located in the soft shale zone, generally at fairly shallow depths. The movements are old and have occurred slowly and intermittently over a period of many centuries. In most areas no movement is discernible at the present time. Considering the mechanics of landslides, as outlined by Terzaghi,(7) it would appear that erosion at the toe, resulting in increased shearing stresses, was a main contributing cause to the initial slides. The slide action, along with creep at the shale surface, produced an opening of the joints. As a result, the entry of water accelerated swelling and chemical weathering with a subsequent drop in cohesion. It is also possible that high water tables in the adjacent upland produced piezometric pressures, particularly in the weathered surface layer of the shale.

The main value of these studies lies in their application to the design and construction of the proposed project. At any point in the vicinity of the site, a safe and stable dam can be constructed using relatively flat side slopes and, where necessary, widening the section to provide a fillet against unstable shale abutments. The main concern was in connection with the stability of deep cuts in the shale, a portion of which would be through soft shale. Strength tests have indicated that flat slopes would be required in the soft shale and while the actual shear tests indicated that relatively steep slopes would be stable in the hard shale zone, there is good reason to believe that its strength will decrease with time. Tunnels in the shale would involve difficult design and construction problems. As a result, a tentative layout has been chosen which provides for conduits through the base of the embankment on the stable shale bench in the R-6 area, thereby eliminating the necessity of deep cuts adjacent to the inlet and the powerhouse. The conduits would be placed on relatively hard uniform shale. The natural slopes of the abutment immediately upstream from the inlet would be flattened to increase the stability.

Acknowledgments

The ideas presented are not entirely those of the writer, but have been gained from discussions with P.F.R.A. engineers,(2, 4) consultants,(3) and others. This applies particularly to geology and air photo interpretation. The writer wishes to gratefully acknowledge the important part played in the studies by A. Casagrande, whose keen

interest and able guidance have been a great inspiration. The comments and suggestions of K. Terzaghi have also been very useful. The assistance of G. W. Parkinson, Design Engineer, J. G. S. McMorine, Project Engineer, and their respective staffs, is also appreciated. The work described was initially supervised by C. F. Ripley and later by A. S. Ringheim; these men and their staff deserve special mention for their keen interest and untiring efforts. The writer would like to thank J. L. Jaspar and P. J. Rivard for assistance in preparing this paper.

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14. Analytical Studies of Panama Canal Slides, by W. V. Binger, Proceedings of Second International Conference of Soil Mechanics and Foundation Engineering, Vol. 2.

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TABLE 1

DESCRIPTION AND PROPERTIES OF BEARPAW SHALE ACCORDING TO ZONE

Results are average or average range.

	Soft	Medium	Hard
Thickness	0 - 20 ft	0 - 20 ft	100 - 200 ft
Weathering	Sometimes - to full thickness	Sometimes - upper portion only	Rarely - top only
Color-Unweathered	Dark grey	Dark grey	Dark grey
Appearance of natural water content	Glossy	Waxy	Dull, waxy when rubbed with knife or finger-nail
Consistency	Erratic - easily remolded under finger pressure	Less erratic - remolded with difficulty Becoming stiffer with depth	Uniform - can be broken in hands with difficulty. Becoming harder with depth
Slickensides and Joints	Closely spaced, surfaces glossy and wet.	Transition	Greater spacing and generally decrease in frequency with depth
Movement or Sliding	Definite signs in some cases	Transition	No definite sign
Water Content Trend	Erratic	Decreasing with depth	Decreasing with depth
Silt and Fine Sand Lenses	Present in all zones		
Grain Size Distribution - Same for three zones	Sand Sizes .05 to 1mm 3% - 35% Silt Sizes .005 to .05 mm 32% - 35% Clay Sizes less than .005 65% - 30%		
Atterberg Limits - See Figure 8			
Water Content	36 - 29%	31 - 25%	27 - 20%
Dry Density lb/cu ft	85 - 95	90 - 97	>97
Relative Consistency	0.7 - 0.9	0.85 - 1.0	0.9 - 1.0

TABLE 1 (Continued)

	Soft	Medium	Hard
Specific Gravity	2.76	2.76	2.76
Degree of Saturation	—	—	96.4%
Permeability ft/min	—	10^{-7} to 10^{-11}	10^{-7} to 10^{-11}
Strength-Deformation *Direct Shear (Quick)			
Apparent Cohesion lb/sq in	15 - 35	35 - 55	55 - 75
Apparent angle of Internal Friction	12°	15°	18°
Unconfined Compression lb/sq in	7 - 80	80 - 150	150 - 400
*Triaxial (Quick) - limited number of tests			
Apparent cohesion lb/sq in	—	31	46
Apparent angle of internal friction	—	25°	29°
Tangent Modulus lb/sq in	7500	10,000	18,000
Consolidation Compression Index at 14 ton/sq ft	.25 - .35	.15 - .25	.05 - .15
Swelling Pressure ton/sq ft	.3 - 1.0	1.0 - 2.0	2.0 - 6.0

*These values are based on the tangent to the envelope at normal stresses less than 200 p.s.i.

TABLE 2

UNCONFINED COMPRESSION RESULTS - T.P. 399 - DEPTH 92 FT.

Specimen Number	Tangent modulus lb/sq in	Unconfined Strength lb/sq in	Water Content	Strain at Failure
1	15,000	256.0	23.6%	2.5%
2	15,200	258.0	23.6%	2.7%
3	15,200	246.0	23.8%	2.7%
4	15,200	254.0	23.2%	2.6%

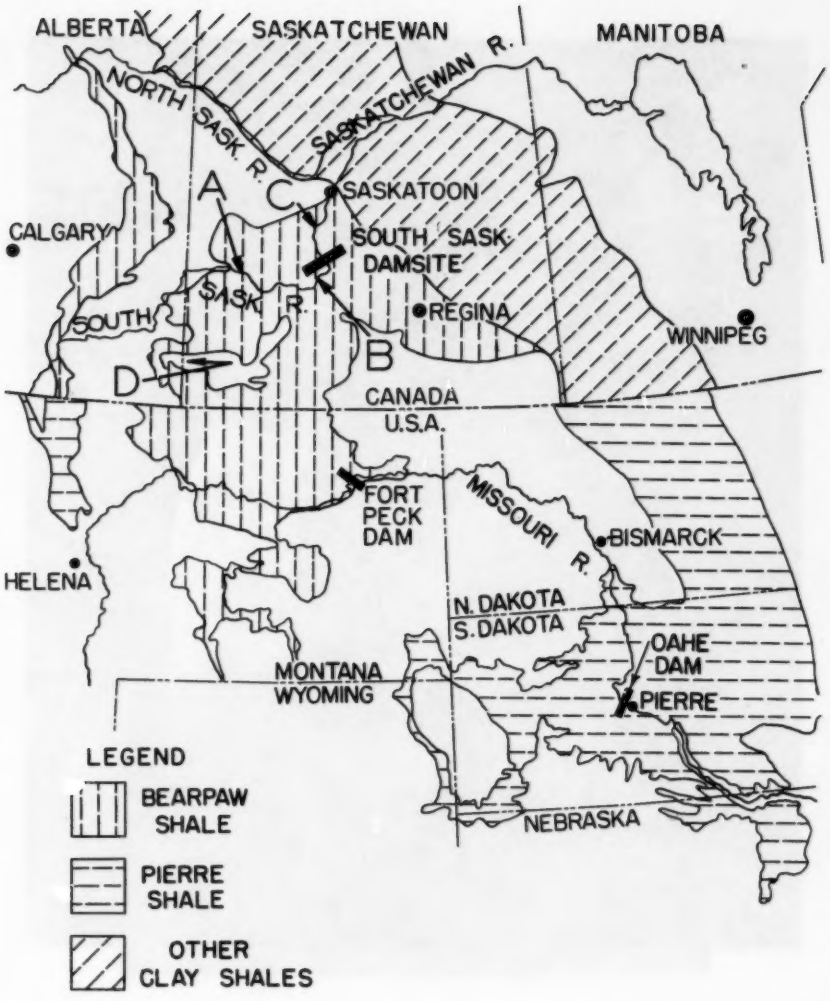


Fig. 1 Regional Map Showing Clay-Shale Areas

Fig. 2 Airphoto of Damsite

Fig. 2A West Abutment at R1 with Drift Portal in Foreground

Fig. 2B West Abutment at R1 with Depression at Right

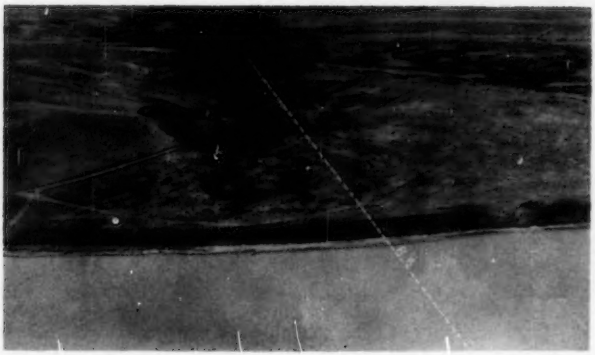
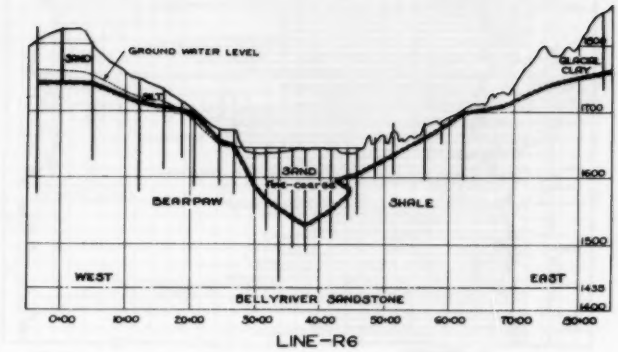


Fig. 2C West Abutment at R6 with Conduit Site in Foreground

Fig. 2D R6 Area in Foreground, R1 in Background, Coteau Ck. At Right



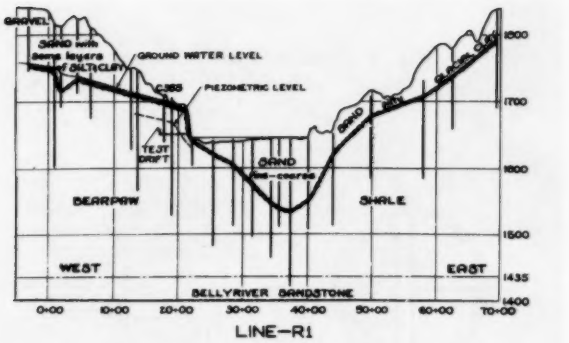


Fig. 3 Soil Profiles of Lines R1 and R6

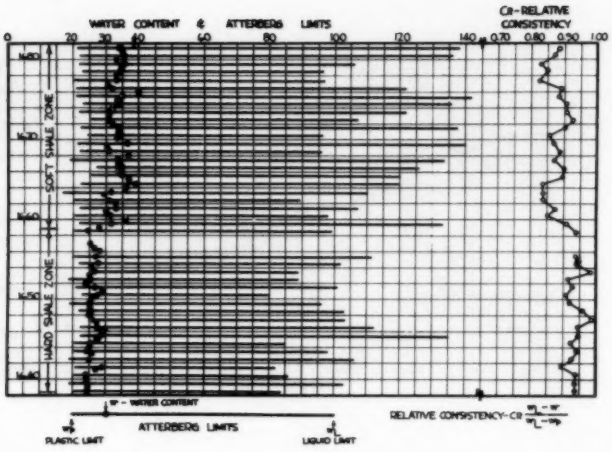


Fig. 4 Profile of Index Properties
Hole C 355

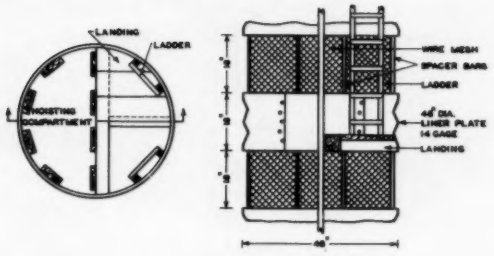


Fig. 5 Test Pit Lining

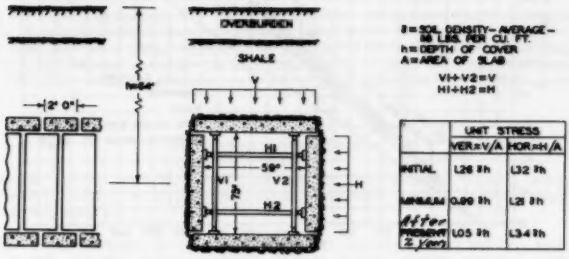


Fig. 6 Pressure Test Section

Fig. 7 Slide Area Showing Subsidence Blocks

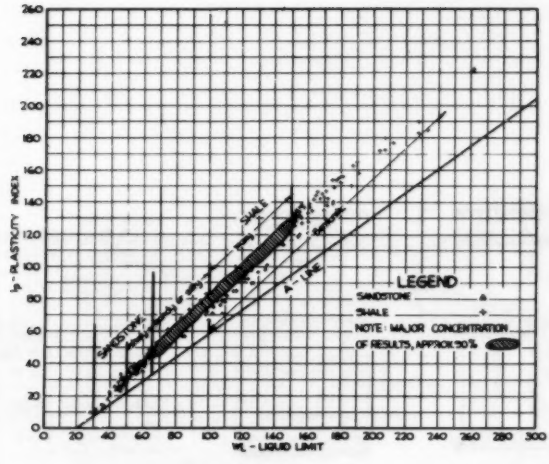


Fig. 8 Plasticity Chart - Bearpaw Shale And Sandstone

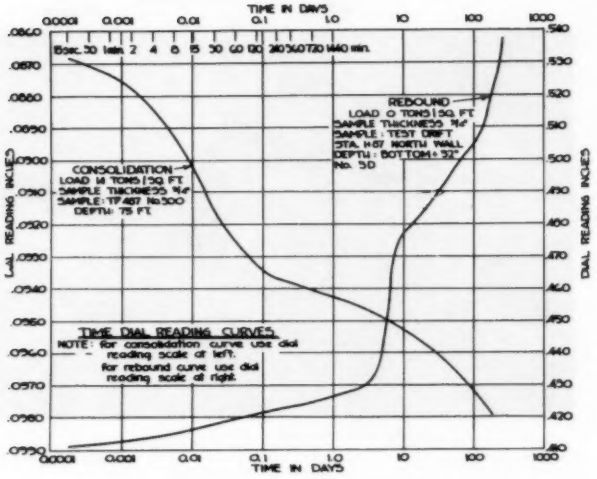

TIME IN DAYS
DIAL READING INCHES
CONSOLIDATION
LOAD 14 TONS / SQ. FT.
SAMPLE THICKNESS 94"
SAMPLE: TP 487 No 500
DEPTH: 75 FT.
REBOUND
LOAD 0 TONS / SQ. FT.
SAMPLE THICKNESS 94"
SAMPLE: TEST DRIFT
STA. 1457 NORTH WALL
DEPTH: BOTTOM = 32'
No. 30
TIME DIAL READING CURVES
NOTE: for consolidation curve use dial reading scale at left.
for rebound curve use dial reading scale at right.

Fig. 9 Time Curves - Consolidation and Swelling

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The technical papers published in the past year are presented below. Technical-division sponsorship is indicated by an abbreviation at the end of each Separate Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

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JUNE: 444(SM)[e], 445(SM)[e], 446(ST)[e], 447(ST)[e], 448(ST)[e], 449(ST)[e], 450(ST)[e], 451(ST)[e], 452(SA)[e], 453(SA)[e], 454(SA)[e], 455(SA)[e], 456(SM)[e].

JULY: 457(AT), 458(AT), 459(AT)[c], 460(IR), 461(IR), 462(IR), 463(IR)[c], 464(PO), 465(PO)[c].

AUGUST: 466(HY), 467(HY), 468(ST), 469(ST), 470(ST), 471(SA), 472(SA), 473(SA), 474(SA), 475(SM), 476(SM), 477(SM), 478(SM)[c], 479(HY)[c], 480(ST)[c], 481(SA)[c], 482(HY), 483(HY).

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a. Presented at the New York (N.Y.) Convention of the Society in October, 1953.
b. Beginning with "Proceedings-Separate No. 290," published in October, 1953, an automatic distribution of papers was inaugurated, as outlined in "Civil Engineering," June, 1953, page 66.
c. Discussion of several papers, grouped by Divisions.
d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
e. Presented at the Atlantic City (N.J.) Convention in June, 1954.

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